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SAFETY OF BRIDGES AND BREAKWATERS ESTIMATED THROUGH GENERAL PRINCIPLES FOR LIMIT STATE DESIGN BY JAPAN SOCIETY OF CIVIL ENGINEERS

Eiichi Watanabe
Kyoto University (Professor Emeritus) and Regional Planning Institute of Osaka (Chairperson), Osaka, Japan
e-mail: watanabe@rpi.or.jp

Although the specifications for Japanese highway bridges are currently under revision in the format of performance-based design, the degree of safety of the designed bridges is not easy to understand. Thus, this paper attempts to disclose the safety and the underlying reliability indices of roadway bridges designed in accordance with the Japanese Specifications for Highway Bridges in the light of a proposal of general principles for the limit state design. The reliability indices and the probability of failure are numerically evaluated assuming the Normal distributions both for the resistance and for the load effect based on the Advanced First-order Second-Moment Method focusing on the relationship between the probability of failure and the factor of safety. Further special reference is made regarding tsunami breakwaters in view of a low frequency but high consequence damages by the Great Eastern Japan Earthquake of March 11, 2011. It was found from the comparison that great discrepancy of the design philosophy and thus the factor of safety exists between the designs of bridges and breakwaters.

Key words: factor of safety, probability of failure, reliability index

1. Introduction

The Great Eastern Japan Earthquake (Tohoku - Pacific Ocean Earthquake) occurred on March 11, 2011 inflicting victims over 20 thousand dead and missing inclusive. The people realized the terror of a low frequency but high consequence disaster and the induced melt-down of nuclear reactors in Fukushima Prefecture afresh. At the same time, most of the Japanese structural engineers must have strongly felt the need for public disclosure of the safety of infrastructures such as bridges, buildings, dikes and tsunami breakwaters that they have built so far. It is really a pity to know that few Japanese engineers understand in reality to what degree bridges, dikes and breakwaters that they have designed and built are safe although they tend to take for granted that most of these infrastructures have been performing quite properly. This anxiety of engineers may be due to so many unknown factors in the design and construction in view of the fact that the Japanese Specifications for Highway Bridges are still in the process toward the complete revision in the format of the performance-based design (JRA 2002).

At the present time, bridges are designed to satisfy either the performance requirement or the specifications. The performance requirement is to satisfy the target performance; whereas the specification requirement is to classify the types and kinds of the structural components so as to satisfy each of the specifications accordingly. However, in the case of the specification requirement, it is unfortunate to say that the target performance is not clearly described. Not only the structural performance but also the safety of bridges against the external loads is not perfectly clear either. From this reason, a subcommittee on the common design code on steel and concrete structures was organized by the Japan Society of Civil Engineers, JSCE (Subcommittee, 1992) to make clear the concept for the limit state design of steel and concrete structures following a design guide for steel structures (JSCE 1987). The present paper is based on the Level I load-factored design method, Level II reliability design method (Melchers, 1987; Stahlbau Handbuch, 1982; Thoft-Christensen and Murotsu, 1986) and the report by this JSCE subcommittee in an attempt to find out in general how quantitatively safe the designed bridges could be when designed accordingly. The required performance of designed bridges can be summarized in the following:

(1) Reasonable safety in the ultimate limit state

(2) Reasonable serviceability and functionality in the serviceability limit state

(3) Reasonable fatigue strength

(4) Easy inspection and monitoring/Reasonable maintenance, management and repair

(5) Environmentally friendly and aesthetically good.

2. Relability index and probability of failure of bridges

What is the safety? One example of the answers to this question may be the "relative freedom from danger, risk, or threat of harm, injury, or loss to personnel and/or property, whether caused deliberately or by accident" (www.businessdictionary.com/definition/safety.html). Then what would be the appropriate index to measure the safety? It may depend on (1) to what degree the safety should be raised and (2) how to improve it? To the first question, although it may at first look desirable to eliminate every possibility of dangerous states and raise the safety to the highest degree, the cost may become astronomically high. Furthermore, people may feel being somehow deprived of their freedom to accomplish the perfect safety. Thus, how to get rid of danger while keeping and enjoying social human life seems to be the problem. Thus, in the human history, people have taken long time to acquire experience to make sure of the safety each time newly developed technology is born.

There may be a wide variety of dangers or failures in bridges. Figures 1a and 1b show symbolically examples of a fail-safe parallel bridge system and a weakest-link series bridge system (Thoft-Christensen and Murotsu, 1986). People feel at ease with the fail-safe system rather than with the weakest-link system because of the inherent redundancy of the former. It must be added, however, that even the fail-safe system may fail just like the weakest-link system due to the aging and deteriorations caused by corrosion, fatigue, excessive natural or artificial forces or by accidents. Once in a while, it may fail progressively in a domino fashion due to accelerated deterioration of performance. Furthermore, there are also great varieties of basic material properties at the ultimate state. Figure 2a shows the ductile fracture model undergoing plastic flow and Fig. 2b the brittle fracture model for example, such as of cable fracture (Thoft-Christensen and Murotsu, 1986). Naturally, the degree of failure may differ considerably depending on whether the material is ductile or brittle. In many cases, the brittle fracture may result in more unfavorable results. Once in a while, local fracture of structures or structural elements may develop into more serious global fracture.

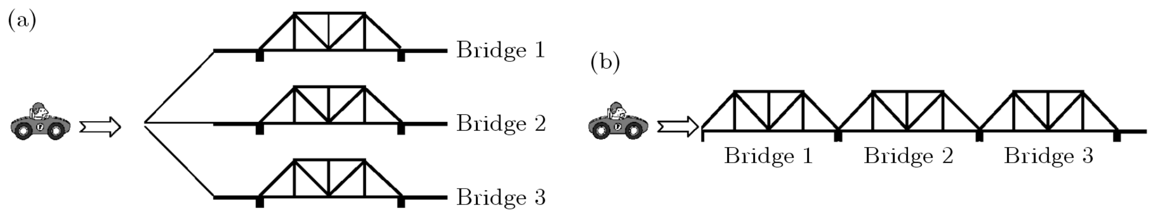


Fig. 1. (a) Fail-safe bridge system and (b) weakest-link bridge system

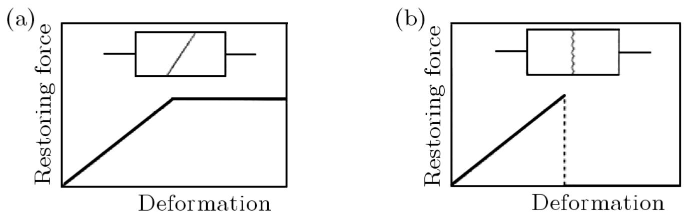


Fig. 2. (a) Ductile fracture and (b) brittle fracture models

Figure 3 shows a list of phenomena resulting in the failure of bridges (Tanaka *et al.*, 2010). Large natural forces include such as typhoons, tornados, earthquakes and tsunamis. The causes of structural defects include such as corrosion, fatigue, buckling, plastic deformation, creep, relaxation and erosion or scouring of bridge piers.

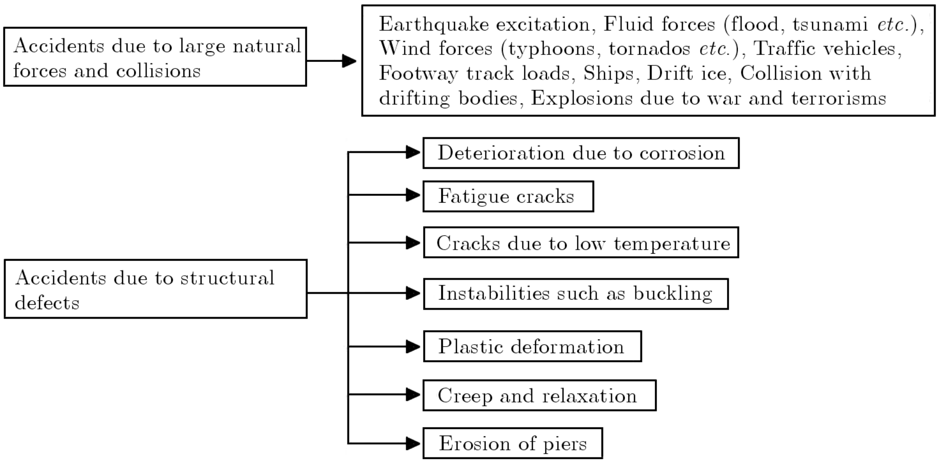
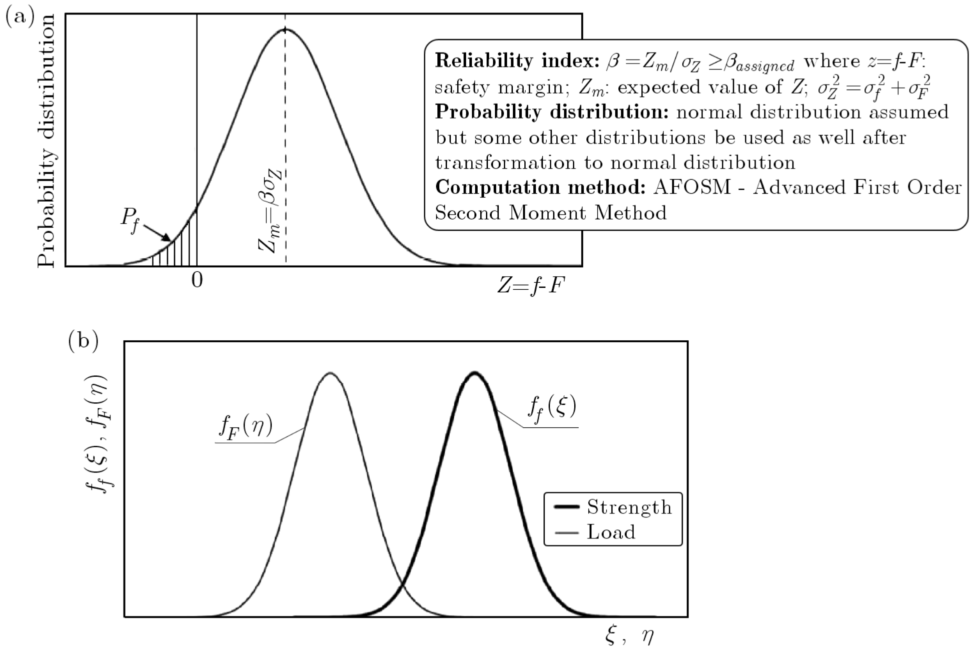


Fig. 3. Phenomena resulting in the failure of bridges

The reliability design methods of structures may be basically classified into 3 groups of Level I, Level II and Level III (Melchers, 1987; Stahlbau Handbuch, 1982; Thoft-Christensen and Murotsu, 1986). Figures 4a and 4b summarize a simple comparison of Level II and Level III reliability design methods, respectively. In this paper, the reliability indices and the corresponding probabilities are evaluated by Level II reliability design method so that the results are reflected in the more realistic and practical method of Level I: Load factored design. The main interest is the evaluation of values of the reliability indices and the corresponding probabilities of failure when the values of the factor of safety, load factor and the resistant factor are varied.

The Level I method as shown in Fig. 5 is used for practical design purposes through calibrations by either Level II or Level III design methods to confirm whether the factor of safety and other safety factors such as the load effect factor γ_F and resistance factors γ_m namely, material resistance factor and structure factor are appropriate. In this paper, the Level II design method is used to evaluate the probability of failure for the sake of simplicity.

Naturally, for the safety of structures, the load effect F should not be greater than the resistance f. In reality, both the load effect F and resistance f have stochastic variations. The stochastic value of load effect F can be confirmed only by observations whereas that of f can be confirmed only by experiments. Furthermore, these stochastic values of F and f are in general described in terms of the expected values, F_m and f_m, respectively. The central factor of safety ν_c is then defined by $\nu_c = f_m/F_m$. The characteristic value for the load effect F can be usually chosen to be F_m. Whereas for the characteristic value of resistance f, some kind of a guaranteed lower limit is naturally expected to be specified to reduce the danger of



$$P_f = \int_{-\infty}^{\infty} f_F(\eta)\, d\eta \int_{-\infty}^{\eta} f_f(\xi)\, d\xi = \int_{-\infty}^{\infty} f_F(\eta) F_f(\eta)\, d\eta \leqslant P_{allowable}$$

$$F_f(\eta) = \int_{-\infty}^{\eta} f_f(\xi)\, d\xi \quad \text{– probability distribution function of resistance}$$

Fig. 4. Comparison between the reliability design methods of Level II and Level III; (a) Level II (normal distribution is assumed on principle), (b) Level III (P_f is evaluated by exact multiple integrals), $f_F(\eta)$ – probability density function of load effect, $f_f(\xi)$ – probability density function of resistance

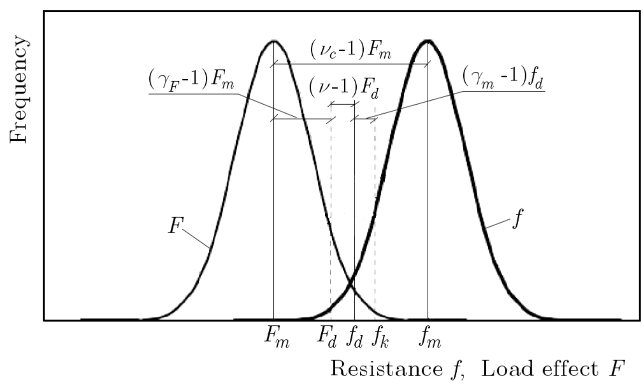


Fig. 5. Reliability design method Level I. Load factor γ_F and material resistance factor γ_m

deficient performance and consequently can be often expressed in the case where the distribution of the resistance f follows the Normal distribution by $f_k = f_m(1 - kV_f)$, where $k = 1.64$ with the probability of non-exceedance of 5% as shown in this figure where σ_f refers to the standard deviation of resistance f and $V_f(= \sigma_f/f_m)$ refers to its coefficient of variation. The design load effect F_d and the design material resistance f_d can be determined respectively by $F_d = F_m\gamma_F$ and $f_d = f_k/\gamma_m$, where γ_F and γ_m are referred to as the load effect factor and the resistance factor, respectively. Then the factor of safety ν is defined by $\nu = f_d/F_d$ (ν can have many values but the value of 1.7 has been used traditionally for bridges) (JSCE 1987). Consequently, the relationship between ν and the central factor of safety ν_c can be shown by $\nu = f_d/F_d = (f_k/\gamma_m)/(F_m\gamma_F) = \nu_c(1-kV_f)/(\gamma_m\gamma_F)$. The load effect factor and the resistance factor are specified in Table 1 and Table 2, respectively (Subcommittee, 1992).

Table 1. Load effect factor γ_F

Limit state	Kinds of load effect	Load effect factor γ_F
Ultimate limit	Permanent load	$1.0 \sim 1.2$ or $1.0 \sim 0.8$ (when smaller value becomes more unfavorable)
	Principal variable load	$1.1 \sim 1.2$
	Secondary variable load	1.0
	Accidental load	1.0
Serviceability limit	All loads	1.0
Fatigue limit	All loads	1.0

Table 2. Resistance factor γ_m

Kind of material	Limit state	Resistance factor γ_m	
Steel $(m=s)$	Ultimate limit	γ_s	$1.0 \sim 1.05$
	Serviceability limit		1.0
	Fatigue limit		$1.0 \sim 1.05$
Concrete $(m=c)$	Ultimate limit	γ_c	1.3
	Serviceability limit		1.0
	Fatigue limit		1.3

According to the Advanced First-Order Second-Moment Method (AFOSM) in the Level II reliability design scheme, assuming the normal distributions for both of the load effect and the resistance, the reliability index β and the probability of failure P_f can be generally expressed by Eq. (2.1) in general or in the case of a low frequency, but a high consequence load effect

$$\beta = \frac{Z_m}{\sigma_Z} = \frac{f_m - F_m}{\sqrt{\sigma_F^2 + \sigma_f^2}} = \frac{\nu_c - 1}{\sqrt{V_F^2 + \nu_c^2 V_f^2}} \qquad P_f = \Phi(-\beta) = 1 - \Phi(\beta) \tag{2.1}$$

In the latter case, $V_F = 0$ and the histogram of the load effect of F in Fig. 5 may be represented by a limited number of isolated scattered spectra of very narrow-banded step functions. In the case of a low frequency but a high consequence load effect

$$\beta = \frac{\nu_c - 1}{\nu_c V_f} \tag{2.2}$$

where β, $\Phi(\beta)$, σ_F and $V_F(= \sigma_F/F_m)$ are referred to as the reliability index, the normalized normal distribution function of β, the standard deviation and the coefficient of variation of the load effect F, respectively.

The probability of failure and the reliability index are shown in Fig. 6 and Fig. 7, respectively for different values of the factor of safety ν in two cases of $V_F = 0.1$; $V_f = 0.05$ and $V_F = 0.2$;

$V_f = 0.1$, respectively. For distributions other than the Normal, references are recommended to be quoted (Thoft-Christensen and Baker, 1982; Thoft-Christensen and Murotsu, 1986).

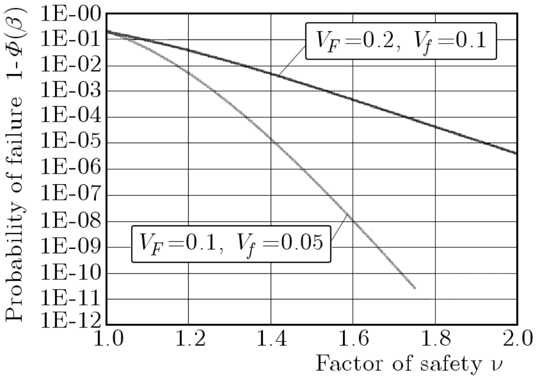


Fig. 6. Probability of failure for different values of the factor of safety ν, when $\gamma_m \gamma_F = 1.0$

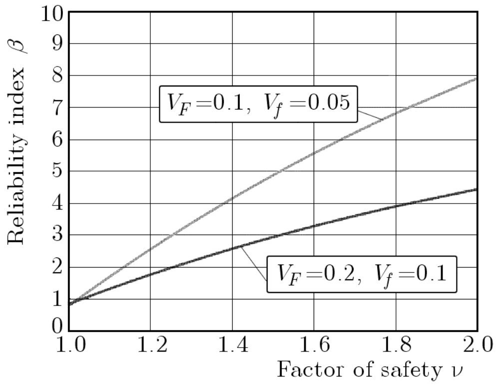


Fig. 7. Reliability Index for different values of the factor of safety ν, when $\gamma_m \gamma_F = 1.0$

3. Determination of structural performance

D. Frangopol and P. Thoft-Christensen showed how the performance of bridges may be reduced by aging on the basis of their research works on concrete bridges in UK studies (Frangopol and Das, 1999; Thoft-Christensen, 1999). Figures 8 and 9 show the general decrement of the value of reliability indices or the performance due to the deteriorations of concrete bridge performances. Particularly, the aging shown in Fig. 9 is really surprisingly large.

They concluded in the following manner:

(1) Reliability index β ranges from 5 to 12 at the initial stage when bridges are newly built (in the case of normal distribution, $P_f = 3.0 \cdot 10^{-7}$ at $\beta = 5$ and $P_f = 0.0$ at $\beta = 12$)

(2) Deterioration process starts at a time point t_I after the service of bridges started (Figs. 8a and 9)

(3) Essential maintenance must be carried out if β decreases below the target value of 4.6 (in the case of normal distribution, $P_f = 2.2 \cdot 10^{-6}$) at a certain time point t_{II}

(4) Preventive maintenance is recommended to be carried out economically well in advance at the time point such as at t_{III} or t_{IV} (Fig. 8b) so that the essential maintenance may not have to be carried out at the time point t_{II}.

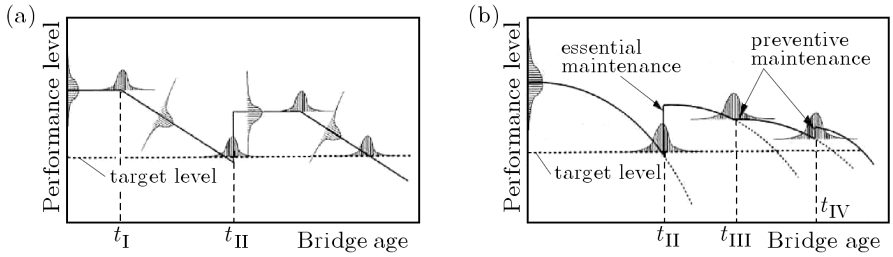


Fig. 8. Deterioration of performance of a concrete bridge; (a) simplified figure (Frangopol and Das, 1999), (b) essential and preventive maintenance (Thoft-Christensen, 1999)

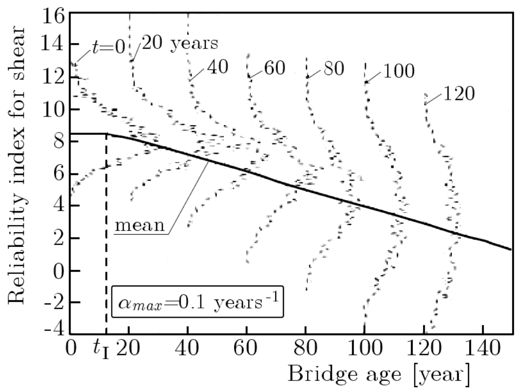


Fig. 9. Time-dependent deterioration process of the reliability index. Note the expected value and stochastic variations – concrete bridge under shear (Frangopol and Das, 1999)

4. Some considerations on the safety of tsunami breakwaters

On March 11, 2011, a great tragedy was inflicted on the North Eastern Japan. The disaster is now officially named the Great Eastern Japan Earthquake. According to the official announcement, the magnitude of the earthquake was Moment Magnitude (MM) 9.0, which is an unprecedented magnitude in Japan. The investigation party was dispatched by JSCE and headed by prof. H. Maruyama (http://committees.jsce.or.jp/report/node/39) (in Japanese). He reported that the main initial cause of the failure of such breakwaters might be the overflow of tsunami over the top of breakwaters. Thus, the overflowed stream could be the initial cause of the scouring of the breakwaters and their turnover. Furthermore, prof. T. Hara of Kohchi University explained the scouring might have occurred in such a way that the overflowed stream turned around just behind the breakwater and washed out the ground. The breakwater becomes unstable due to this scouring, with the buoyancy force acting on the breakwater and the risen water level pushed it from the ocean side to inside resulting in an easier turnover of the breakwater.

Figure 10 shows the estimated tsunami travel time forecast map for the 2011 Great Eastern Japan (hereafter referred to as GEJ for convenience) Earthquake from the U.S. NOAA. Moreover, Figs. 11a-c show some pictures of a piled-up ship over a breakwater and turnover of breakwaters by the courtesy of Mr. I. Teranishi who surveyed damages at several places badly failed during this earthquake as a member of the Japan Bridge Association.

Figure 12 classifies the failures of breakwaters by the Asia-Pacific Center for Coastal Disaster Research of the Port and Airport Research Institute (www.bousai.go.jp/jishin/chubou/higashinihon/4/2-2.pdf in Japanese).

As already mentioned, JSCE has reported that the main initial cause of the failure of such breakwaters might be the overflow of tsunami over the top of breakwaters as shown in Fig. 12d: overflowed stream resulting in the scouring of the ground just behind and inside the breakwater and the subsequent turnover. After literature survey, the factor of safety of the breakwater in

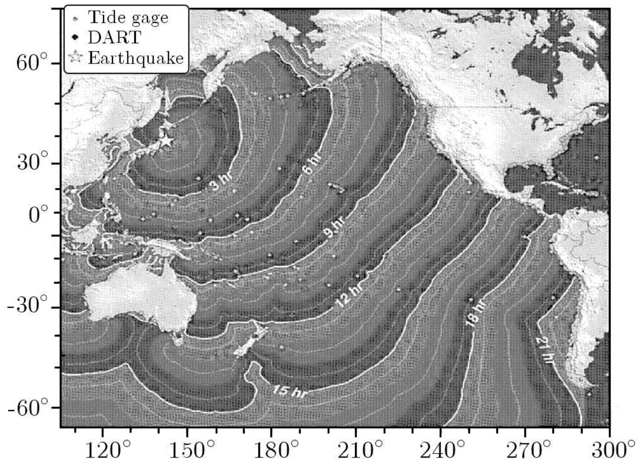


Fig. 10. Estimated tsunami travel time forecast map for the 2011 GEJ Earthquake from the U.S. NOAA (http://en.wikipedia.org/wiki/File:2011Sendai-NOAA-TravelTime-Ttvulhvpd9-06.jpg)

Fig. 11. (a) Piled-up ship on bank at Kamaishi Harbor, (b) breakwaters turned over counter-clockwise at Yamada-Cho, (c) breakwaters turned over showing bottoms at Yamada-Cho (courtesy of Mr. I. Teranishi)

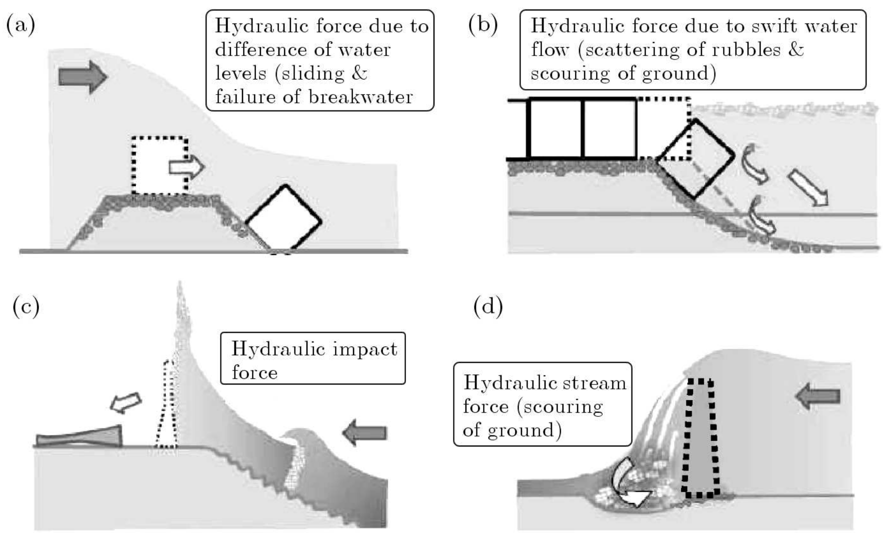


Fig. 12. Classification of the failures of tsunami breakwaters (courtesy of Mr. S. Takahashi of the Port and Airport Research Institute)

Japan was found to be only 1.0 and that of the walls against the sliding is only 1.2. These low factors of safety are very much surprising to us, structural engineers who are accustomed to the factor of safety as high as about 1.7 (JSCE 1987). It has been found that one of the largest tsunamis in Japan in the past with the resemblance to the 11 March 2011 tsunami would be the one which occurred at Johgan Earthquake of 869 A.D. Thus, the return period of this size of tsunami may be considered to be once in every 1000 years. Keeping this in mind, the probability of overflow, namely, the probability tsunami exceeding the height of breakwater is evaluated for different values of the factor of safety.

Tsunami hazard curves are often used to designate the relationship between the probability of exceedance of occurrence and the height of tsunami. Needless to say, these curves generally depend on many factors such as the topography of the surrounding sea, locations of active faults, epicenter and magnitude of earthquake, starting point of asperity and non-uniformity of the faults. For example, Annaka *et al.* developed the logic-tree models for local tsunami sources around Japan and for distant tsunami sources along the South American subduction zones. Logic-trees were made for tsunami source zones, size and frequency of tsunamigenic earthquakes, fault models, and standard error of estimated tsunami heights (Annaka *et al.*, 2007). In this paper, however, since the author is not an expert in this area, the discussions are made presumptuously in a very simple manner on the assumption that tsunami can occur anywhere near the sea zones potentially susceptible to tsunami. A simplified relationship between the expected tsunami height H and the return period of tsunami T may be expressed satisfactorily quite well by Eq. (4.1) and Fig. 13b as an approximation of the hazard curve, Fig. 13a by Yoshida *et al.* (2007)

$$\frac{H}{H_0} = \frac{\log T}{\log T_0} = \frac{\log(1/T)}{\log(1/T_0)} \tag{4.1}$$

where T_0 corresponds to the return period of the maximum tsunami height H_0. From evidences from investigations such as geological excavations, the Johgan Earthquake is thought to be very close to March 11, 2011 GEJ Earthquake. The Johgan tsunami occurred in AD 869 with the earthquake of an estimated magnitude of M8.3 or more. Thus, it may be conveniently assumed that $T_0 = 800$ years and $H_0 = 30\,\mathrm{m}$ (max tsunami height recorded on March 11, 2011) as shown in Table 3.

Table 3. Prediction of tsunami height in reference to March 11, 2011 earthquake and Johgan earthquake

T [Year]	800	100	50	30	10
H [m]	30	20.66	17.6	15.3	10.3

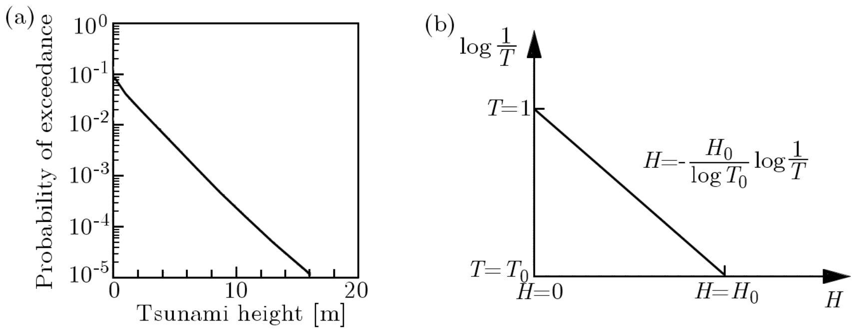


Fig. 13. Relationship between $\log(1/T)$ and tsunami height H; (a) hazard curve by Yoshida (2007), (b) hazard curve, Eq. (4.1)

Since T_0 refers to the return period, the probability of tsunami occurrence at least once in consecutive Q years P_r may be expressed by Eq. (4.2)

$$P_r = 1 - \Big(1 - \frac{1}{T_0}\Big)^Q \tag{4.2}$$

Thus, the probability P_r of tsunami occurrence can be given as in Fig. 14 and in Table 4. It may be observed that naturally, the longer the value of Q, the larger the probability P_r becomes. It would be very important to know from Table 4 and Fig. 14 that when the period Q is close to the target return period T_0, the probability of occurrence of the tsunami is something like 63% while it is about 10% when the period Q is 1/10 of the return period T_0.

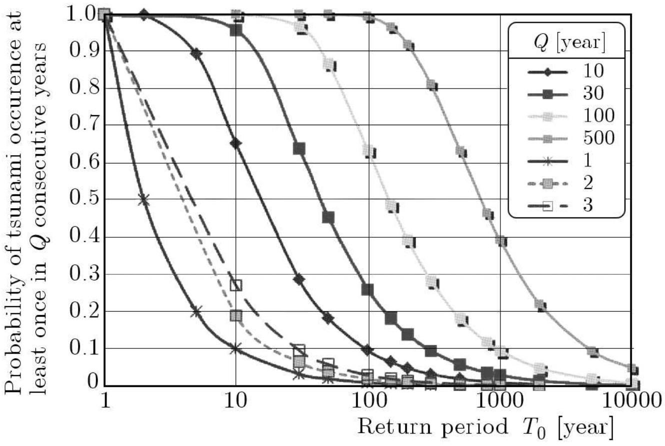


Fig. 14. Probability P_r of tsunami occurrence such as March 11, 2011 at least once in consecutive Q years

Table 4. Probability P_r of tsunami occurrence such as March 11, 2011 at least once in consecutive Q years

Q [Year]	10	30	100	500	800
P_r	0.012	0.037	0.118	0.465	0.632

So far, the discussion was made on the probability of occurrence of tsunami, and this has only a little bearing on the safety of tsunami breakwater. Now, let us next consider the reliability and safety of the tsunami breakwaters. Let d represent the height of a breakwater, then the safety margin Z may be expressed by

$$Z = d - H \geqslant 0 \tag{4.3}$$

Again by applying the AFOSM, the reliability index β can be expressed by

$$\begin{aligned}\beta &= \frac{\mu_Z}{\sigma_Z} = \frac{\mu_d - \mu_H}{\sigma_Z} = \frac{\mu_d - \mu_H}{\sqrt{\sigma_d^2 + \sigma_H^2}} = \frac{\frac{\mu_d}{\mu_H} - 1}{\sqrt{\frac{\sigma_d^2}{\mu_H^2} + \frac{\sigma_H^2}{\mu_H^2}}} = \frac{\nu_d - 1}{\frac{\sigma_H}{\mu_H}\sqrt{\frac{\sigma_d^2}{\sigma_H^2} + 1}} \\ &= \frac{\nu_d - 1}{V_H\sqrt{\frac{\sigma_d^2}{\sigma_H^2} + 1}} = \frac{\nu_d - 1}{\sqrt{V_d^2\nu_d^2 + V_H^2}}\end{aligned} \tag{4.4}$$

where $\nu_d = \mu_d/\mu_H$ represents the factor of safety of a breakwater; μ_Z, μ_d, μ_H and σ_Z, σ_d, σ_H represent the expected values and the standard deviations of Z, d, H, respectively. Furthermore, V_d and V_H designate the coefficients of variation of d and H, respectively. Now, upon differentiation of Eq. (4.1), Eq. (4.5) can be obtained

$$\frac{dH}{H_0} = \frac{1}{\log T_0}\frac{dT}{T} \tag{4.5}$$

From Eq. (4.5), the relationship between σ_H and σ_T of H and T, respectively can be found as

$$\sigma_H^2 = \Big(\frac{H_0}{T \log T_0}\Big)^2 \sigma_T^2 \tag{4.6}$$

Thus, the reliability index can be expressed by Eq. (4.7) evaluated at the critical design point H_0 and T_0 as

$$\beta = \frac{\nu_d - 1}{\sqrt{V_d^2 \nu_d^2 + V_H^2}} = \frac{\nu_d - 1}{\sqrt{V_d^2 \nu_d^2 + \Big(\frac{V_T}{\log T_0}\Big)^2}} \tag{4.7}$$

where V_T represents the coefficient of variation of T. Then the reliability index β and the probability P_f of tsunami height exceeding the height of the breakwater is shown by Fig. 15 and Fig. 16, respectively. Furthermore, under condition of $V_T = 0.2$ and $V_d = 0.1$, the probability of failure (overflow) P_f is obtained as shown in Fig. 17 when the return period of tsunami T_0 and the predicted maximum tsunami height H_0 are varied. Just as for the structural reliability problems, in the case of a low frequency but high consequence occurrence of a huge tsunami, the reliability index becomes by neglecting the coefficients of variation V_H and V_T

$$\beta = \frac{\nu_d - 1}{\nu_d V_d} \tag{4.8}$$

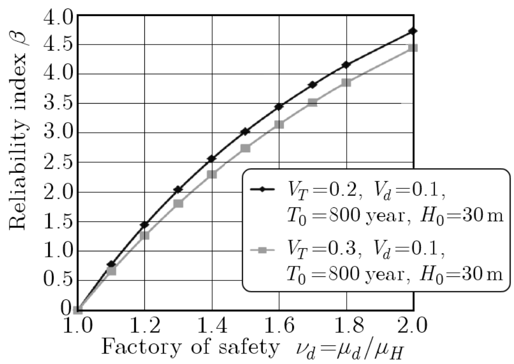


Fig. 15. Reliability index of breakwater

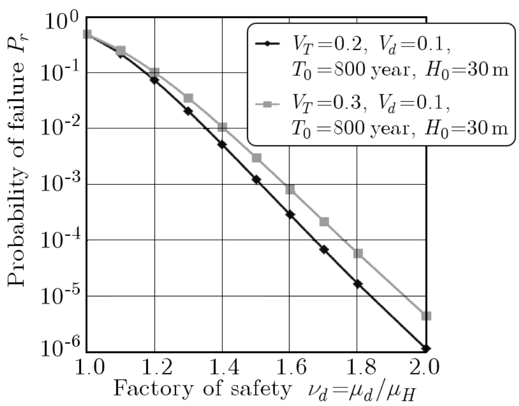


Fig. 16. Probability of tsunami exceeding height of the breakwater

It is surprising to know that according to the current design method of a breakwater against tsunami, since the factor of safety is very close to one, the probability of failure, namely tsunami exceeding the height of the breakwater is 50%. Even if the factor of safety is improved to be 1.2, the probability of failure may be something like 10%.

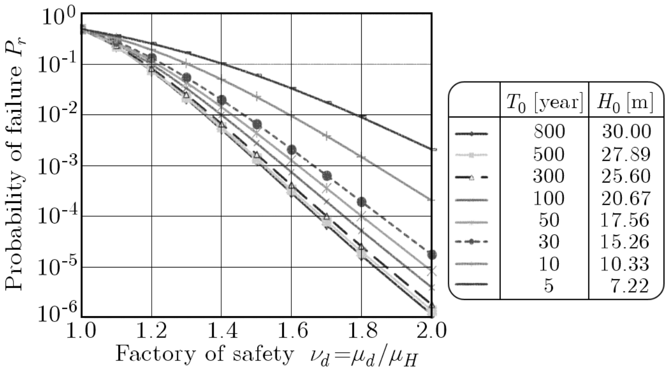


Fig. 17. Change of probability of failure P_f when the return period T_0 is varied ($V_T = 0.2$ and $V_d = 0.1$)

5. Conclusions

The current Japanese Specifications for highway bridges are under revision in the format of performance-based design in the genuine sense although there may be more detailed laborious efforts yet to be made by the writers of the Specifications such as time-taking detailed calibrations for practical purposes. Thus, attempts are made in this paper to roughly evaluate the general reliability with special emphasis on the reliability index and probability of failure of the bridge design. Special attempts are made also to evaluate the reliability of tsunami breakwaters. It is found that the current practice of the design of breakwater is such that the factor of safety of tsunami height exceeding the height of the breakwater is just one. In view of the fact that the overflow of tsunami over the top of the breakwater eventually results in the fatal catastrophe of the scouring, sliding or turning over of breakwaters, the review or reexamination of the present factor of safety of the breakwater may become worthwhile in recognition of larger factors of safety, 1.7, of bridge design employed at present in preparation of the future low frequency but high consequence tsunami in the near future. As a matter of fact, Japan is threatened with a linked series of huge earthquakes of Tokai (East Sea), Tonankai (South-East Sea) and Nankai (South Sea). These linked earthquakes in a domino reaction are predicted to occur at any moment immediately near future in the similar scale of the Great Eastern Japan Earthquake. Finally, how to be prepared for natural disasters of a low frequency but high consequences is a great challenge not only for the Japanese people but for all the people in the world, and for this purpose people of the world should work together and make effort to find the best solution for their protection.

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Szacowanie bezpieczeństwa mostów i falochronów na podstawie ogólnych zasad przyjętych przez Japońskie Stowarzyszenie Inżynierów Lądowych dla konstrukcji narażonych na graniczny stan obciążenia

Streszczenie

Mimo, że zalecenia projektowe dla mostów drogowych budowanych w Japonii są właśnie modyfikowane pod kątem właściwości eksploatacyjnych, szacowanie stopnia bezpieczeństwa takich konstrukcji wciąż pozostaje problemem nie do końca zrozumiałym jednoznacznie. W prezentowanej pracy przedstawiono próbę przybliżenia zagadnienia bezpieczeństwa i wskaźników niezawodności dla mostów projektowanych zgodnie z wytycznymi ujętymi w Japońskich Specyfikacjach Dla Mostów Drogowych. Regulacje te definiują ogólne zasady stosowane w konstrukcjach narażonych na eksploatację w granicznym stanie obciążenia. Wskaźniki niezawodności i prawdopodobieństwo uszkodzenia oszacowano numerycznie, przyjmując normalny rozkład wytrzymałości na obciążenie. Do obliczeń wykorzystano metodę momentów pierwszego i drugiego rzędu dla zależności pomiędzy prawdopodobieństwem zniszczenia i wskaźnikiem bezpieczeństwa. W dalszej części odniesiono się do problemu falochronów zabezpieczających przed tsunami jako konstrukcji narażonych na rzadko występujące, lecz groźne w konsekwencjach obciążenia. Dyskusję przeprowadzono w kontekście Wielkiego Wschodnio-Japońskiego Trzęsienia Ziemi z 11 marca 2011 roku. Z analizy porównawczej jasno wynikło, że istnieją ogromne rozbieżności w filozofii projektowania mostów drogowych i falochronów i tym samym różna ocena wskaźników bezpieczeństwa tego typu konstrukcji.

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